AD-A035 763 CONSTRUCTION ENGINEERING RESEARCH LAB (ARMY) CHAMPAI--ETC F/G 13/13
DESIGN CRITERIA FOR THEATER OF OPERATIONS STFEL HIGHWAY BRIDGES--ETC(U)
JAN 77 L I KNAB, W H MUNSE, A H ANG

UNCLASSIFIED CERL-TR-M-195-VOL-1 NL

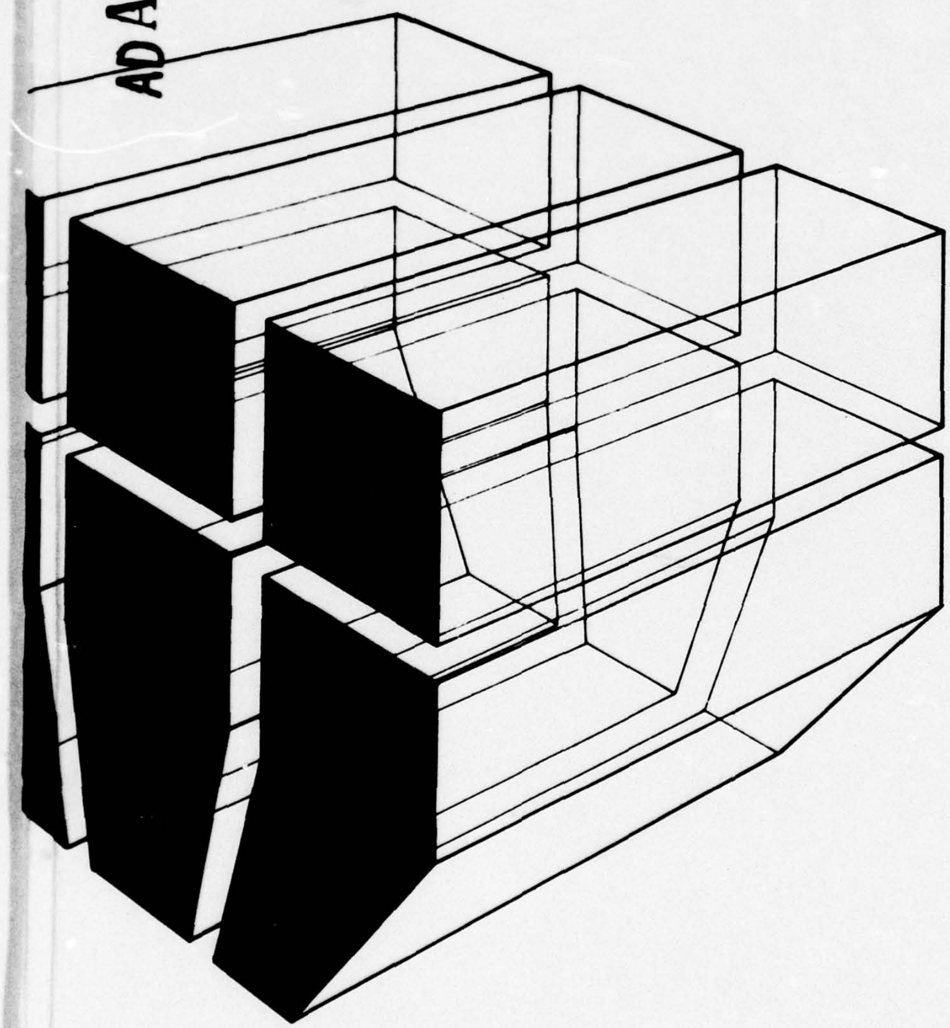
1 OF 1
ADA035763

END
DATE
FILMED
3-77

construction
engineering
research
laboratory

TECHNICAL REPORT M-195
January 1977
AFCS Design Parameters for T/O Material Applications

DESIGN CRITERIA FOR THEATER OF OPERATIONS STEEL HIGHWAY BRIDGES
VOLUME I



by
L. I. Knab
W. H. Munse
A. H-S. Ang
S. T. Rolfe
W. W. Sanders, Jr.
H. A. Elleby
T. V. Galambos

Approved for public release; distribution unlimited.

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER (14) CERL-TR-M-195-Vol-1	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER (9)
4. TITLE (and Subtitle) (6) DESIGN CRITERIA FOR THEATER OF OPERATIONS STEEL HIGHWAY BRIDGES, VOLUME I.		5. TYPE OF REPORT & PERIOD COVERED FINAL rept., 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) (10) L. I. Knab, S. T. Rolfe, T. V. Galambos W. H. Munse, W. W. Sanders, Jr A. H-S. Ang, H. A. Elleby		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS CONSTRUCTION ENGINEERING RESEARCH LABORATORY P. O. Box 4005 Champaign, IL 61820		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS (16) 4A763734DT34-~~04-002~~
11. CONTROLLING OFFICE NAME AND ADDRESS (17) Φ4		12. REPORT DATE (11) Janu[redacted]77 13. NUMBER OF PAGES 33
14. MONITORING AGENCY NAME & ADDRESS(If different from Controlling Office) (12) 35p.		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

Copies are obtainable from National Technical Information Service
Springfield, VA 22151

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

steel highway bridges
design criteria
theater of operations temporary bridges

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report presents structural design criteria recommendations for theater of operations (T/O) temporary steel highway bridges. The report consists of two volumes: Volume I provides the design criteria, procedures, and material specifications for both Army Facilities Components System (AFCS) bridges and bridges designed by combat engineers; Volume II contains the development and justification for the criteria in Volume I.

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

405 279

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

1/B

Block 20 continued.

Evaluation of the existing military design criteria indicated that their use can result in bridges which are potentially unsafe or perform poorly. State-of-the-art methodologies were used to develop static, fatigue, and brittle fracture criteria which should result in improved safety and performance for T/O bridges. The recommended criteria are based on modifications to existing criteria for military and permanent nonmilitary bridges. The AFCS bridge criteria are intended for use by engineers in bridge design offices. Use of the criteria by combat engineers to design bridges in the field requires that the recommended criteria be incorporated in existing Army technical and field manuals.

When the criteria recommended in this report are used, the material weight for steel stringers in T/O bridges will typically be from 15 to 20 percent less than when permanent bridge criteria are used.

FOREWORD

This study was conducted for the Directorate of Facilities Engineering, Office of the Chief of Engineers (OCE), under Project 4A763734DT34, "Development of Engineer Support to the Field Army"; Task 04, "Base Development"; Work Unit 002, "AFCS Design Parameters for T/O Material Applications."

The OCE Technical Monitor is Mr. R. H. Barnard.

The work was performed by the Construction Materials Branch (MSC) of the Materials and Science Division (MS), U.S. Army Construction Engineering Research Laboratory (CERL). Dr. L. I. Knab was the CERL Principal Investigator for the project. Mr. P. A. Howdyshell is Chief of MSC and Dr. G. L. Williamson is Chief of MS.

Contractors contributing to the study were W. H. Munse and A. H-S. Ang of the University of Illinois at Urbana-Champaign, Urbana, IL; S. T. Rolfe of the University of Kansas, Lawrence, KS; W. W. Sanders, Jr. and H. A. Elleby of Iowa State University, Ames, IA; and T. V. Galambos of Washington University, St. Louis, MO.

COL J. E. Hays is Commander and Director of CERL and Dr. L. R. Shaffer is Technical Director.

CONTENTS

CONTENTS

TABLES

FIGURES

DESIGN CRITERIA FOR THEATER OF OPERATIONS STEEL HIGHWAY BRIDGES

VOLUME I

1 INTRODUCTION

Background

Engineer troop units in a theater of operations (T/O) are assigned diverse construction missions which often have critical completion date requirements. These missions often involve erecting or assembling standard prefabricated components, such as those available for buildings and tactical bridges, or constructing structures using conventional materials and components.

The Army Facilities Components System (AFCS) contains standard designs for many of the structures that T/O troop units may be required to construct. The AFCS Technical Manual (TM) 5-302[1] and its related logistical references TM 5-301[2] and TM 5-303,[3] contain many standard structural designs which use both standard and prefabricated components. These manuals, which also contain bills of materials and construction practice guidance, greatly reduce the amount of planning and design effort required to complete a construction assignment.

However, T/O construction for which prefabricated materials and/or standard designs are not available is also required. Engineers in the field must plan and design such structures for particular missions using as guidance such references as TM 5-312,[4] FM 5-35,[5] TM 5-744,[6] and TM 5-882-2.[7]

Since the design life of T/O structures is typically from 2 to 5 years, use of civilian structural design codes, which are generally based on design lives of 50 to 75 years and relatively high levels of reliability, can result in unnecessarily conservative and uneconomical designs for the limited needs of T/O structures. In addition, environmental conditions which can adversely affect the performance of structural materials, such as low temperatures in arctic regions, should be taken into account.

Purpose

The purpose of this report is (1) to develop appropriate reliability models and evaluate the reliability levels and consistency of current military structural steel design criteria for T/O bridges in light of their 2 to 5 year design life and environment, and (2) to formulate, in a simple format, recommendations for design criteria for T/O structural steel bridge members and fasteners, based on reliability concepts.

Approach

Appropriate reliability models were developed to analyze and evaluate the safety levels and consistency of existing structural design criteria for steel (rolled shapes) members and fasteners for bridges. The models were then used to develop static, fatigue, and brittle fracture criteria for T/O bridge members and fasteners subjected to dead plus live loads.

Chapter 2 summarizes the development of these design criteria. (Appendices A through I in Volume II describe and justify the design criteria development and recommendations in detail.) Chapter 3 presents the recommended design criteria for AFCS bridges designed by bridge design offices. Chapter 4 provides the recommended design criteria for bridges designed in the field by combat engineers. Chapter 5 provides an example of the use of the criteria presented in Chapters 3 and 4. Chapter 6 evaluates the existing TM 5-312 criteria for the static load case, and Chapter 7 presents the study's conclusions and recommendations.

Technology Transfer

The research in this report impacts on TM 5-312, FM 5-34,[8] FM 5-35, and TM 5-301, -302, and -303.

[1] *Army Facilities Components System (AFCS)—Designs*, TM 5-302 (Department of the Army, September 1973).

[2] *Army Facilities Components System (AFCS)—Planning*, TM 5-301 (Department of the Army, September 1973).

[3] *Army Facilities Components System (AFCS)—Logistic Data and Bills of Materials*, TM 5-303 (Department of the Army, September 1973).

[4] *Military Fixed Bridges*, TM 5-312 (Department of the Army, December 1969), with changes 1 and 2.

[5] *Engineers' Reference and Logistical Data*, FM 5-35 (Department of the Army, April 1971).

[6] *Structural Steelwork*, TM 5-744 (Department of the Army, October 1968).

[7] *Structural Design—Emergency Construction*, TM 5-882-2 (Department of the Army, August 1963).

[8] *Engineer Field Data*, FM 5-34 (Department of the Army, December 1969).

2 SUMMARY OF DESIGN CRITERIA DEVELOPMENT

The report provides a rational basis for developing consistent design criteria for short-life structures. A consistent design is one in which the probability or chance of reaching a limit state (limit state probability) of a structural element (member, fastener, or connection) is compatible with the other limit state probabilities of that element or other elements in the structure.

For the static failure mode, member resistance and structural loading data were combined to determine existing and recommended limit state probabilities and their corresponding safety index values. Best estimates of central tendency (average values) coupled with measures of dispersion (coefficients of variation) of the resistance and load were used to compute the safety index values using first order reliability theory. These safety index values, which were computed for permanent and temporary structural elements, were used as guides in determining recommended allowable stresses. Recommended criteria for the static failure mode for temporary bridges were determined so as to yield safety index values which provide adequate safety relative to that of past temporary and permanent bridges, while reflecting the bridges' temporary nature.

For the fatigue failure mode, the limit state probability of surviving a given number of fatigue load cycles was calculated. This limit state probability was compared for both permanent and temporary bridges and used as a basis for developing the fatigue allowable stress ranges. The recommended T/O fatigue allowable stress ranges should provide adequate safety and performance relative to permanent bridges while accounting for the temporary nature of the T/O bridges. The limit state probabilities and corresponding allowable stress ranges were developed based on: (1) the fatigue resistance of details at which fatigue may control the design, and (2) the stress ranges or loading frequency history and the number of cycles of loading to which these details are subjected.

Brittle fracture, an unstable crack growth failure, which usually occurs at low temperatures, is covered by increasing the material toughness requirements and/or reducing the allowable static tensile stress.

The criteria development assumed that adequate fabrication, assembly, and erection procedures and practices are used; special provisions and criteria are recommended when lower quality fabrication, assembly, and erection practices are likely to occur.

Significant materials and labor savings can result from using the recommended T/O bridge criteria rather than permanent bridge criteria. (See **Safety, Savings, and Cost Effectiveness** in Chapter 3.)

3 RECOMMENDED DESIGN CRITERIA FOR AFCS BRIDGES

Scope

The criteria presented in this chapter are recommended for use in designing T/O steel girder and truss-type bridges* which have design lives of 2 to 5 years and are fabricated from hot-rolled steel elements. The maximum deviation from the recommended criteria shall not exceed 2 percent (on the unconservative side) of the indicated design requirements.

The criteria apply to hot-rolled and fabricated (built-up) shapes, including standard wide flange shapes as well as plate girders, trusses, and appropriate fasteners and connections. These criteria do not cover composite girders, hybrid girders, box girders, tapered members, members subjected to combined bending and axial force, biaxial bending, torsion, doubly nonsymmetric shapes, and friction joints.

Unless otherwise noted, all references to the American Association of State Highway and Transportation Officials (AASHTO)[9] and the American Institute of Steel Construction (AISC)[10] refer to their 1973 and 1969 specifications, respectively. The

*Girder design shall be subject to the Association of State Highway and Transportation Officials (AASHTO) requirements of Articles 1.7.68 through 1.7.74 except as herein modified; truss design shall be subject to the AASHTO requirements of Articles 1.7.75 through 1.7.89 except as herein modified. In Articles 1.7.69 and 1.7.88, the values of the limits of compressive bending stress or axial compressive stress specified as 0.55 of the minimum specified yield stress F_y shall be taken as 0.66 F_y.

[9] *Standard Specifications for Highway Bridges*, 11th ed. (American Association of State Highway and Transportation Officials [AASHTO], 1973).

[10] *Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings* (American Institute of Steel Construction [AISC], 1969), with supplements 1 (1970), 2 (1971), and 3 (1974).

grades of steel to which the criteria in this chapter apply are those covered by the AASHTO specification, except that the specified minimum steel yield point shall not exceed 50 ksi.*

Member and fastener criteria are provided for the dead plus live load (vehicle weight plus payload) case and the associated impact effects. Other loads, including wind, water, ice, and earthquake, are not covered.

Appropriate criteria for permanent bridge structures, such as AASHTO or AISC, can be used for steel members and fasteners in temporary bridges not covered by the criteria recommended herein, provided the recommendations in the **Loads, Moments, and Forces** section of this chapter are used.**

For permanent military bridges, the most recent AASHTO requirements are recommended, provided the AASHTO requirements are used with the recommendations in the **Loads, Moments, and Forces** section of this chapter.**

The recommended criteria apply to highway bridges and do not cover railroad bridges.

Safety, Savings, and Cost Effectiveness

In the past, increased allowable stresses for static loading have been permitted for temporary bridges (TM 5-312). The safety levels† corresponding to the allowable stresses and/or design procedures in TM 5-312 for temporary bridges were evaluated and used to determine recommended design criteria which reflect the temporary nature of T/O bridges. Hence, the recommended criteria should result in temporary bridges which are at least as safe against static collapse as temporary bridges constructed using TM 5-312 allowable stresses and procedures. The safety levels corresponding to the recommended criteria for T/O bridges loaded with their normal or maximum loads are comparable to or exceed the safety levels corresponding to permanent civilian bridges loaded with their maximum loads, but are lower than the safety levels corresponding to permanent bridges loaded with their normal loads. Thus, the recommended criteria should result in acceptably safe temporary bridges when compared with military temporary bridges of the past and current permanent civilian bridges loaded with their maximum loads.

This determination of safety was based on the assumption that fabrication, assembly, and erection practices are adequate; if this assumption is not valid, the bridge's safety can be significantly reduced. (See the **Fabrication, Assembly, and Erection** section of this chapter.)

When criteria recommended in this chapter for T/O bridges are used, material weight for stringers (standard rolled shapes under static load) will typically be from 15 to 20 percent less than when permanent bridge (AASHTO) allowable stress criteria are used with the design procedures recommended in this chapter. Stringer weights obtained using the recommended criteria compare to those obtained using existing TM 5-312 criteria as follows:

1. Similar weights are obtained when concrete or glued-laminated timber decks are used in environments which are not cold.

2. Slightly increased weights are obtained when using the recommended criteria if nailed-laminated timber, plank, or multiple-layered plank decks are used in environments which are not cold.

3. Significantly increased weights are obtained when using the recommended criteria for bridges in cold environments.

Weight savings resulting from using the criteria recommended in this chapter rather than using permanent (AASHTO) bridge allowable stresses with the recommended design procedures in this chapter are 0 to 10 percent for axially loaded compression members (typical) and 33 percent for tension members.

In certain cases, the recommended allowable stresses for fasteners exceed the permanent (AASHTO and AISC) stresses for fasteners. These increased allowable stresses will save labor during fabrication and erection by reducing the number of bolts and bolt holes required.

Use of the recommended T/O criteria to prevent static failure will be cost effective if the material,

*SI conversion factors for all units of measure used in this report are given at the end of the report.

**In certain cases, unrealistically low loads, moments, and forces can result if the current procedures in TM 5-312 are used.

†See Appendices A through D of Volume II of this report for details of the safety analyses.

transportation, labor, and storage cost savings exceed the cost of implementing the criteria (see the **Implementation** section of this chapter). Use of the recommended criteria appears reasonable, provided that the temporary bridges used in the past have been acceptably safe and have performed satisfactorily. The decision to use the recommended criteria is to be made by the AFCS Office at OCE (DAEN-FEE-A).

Past design criteria and procedures for temporary bridges have not required consideration of fatigue failure or brittle fracture failure (failure due to unstable crack propagation which usually occurs at cold temperatures). Hence, inclusion of fatigue and brittle fracture failure in the design process should increase the safety of the temporary bridges, provided the failure mode is by fatigue or brittle fracture. Use of the fatigue and brittle fracture criteria will be cost effective if the savings which result from the increased safety exceed the additional labor, including implementation (see the **Implementation** section of this chapter), and material costs incurred.

It should be noted that there recently has been a growing awareness of the problems associated with both fatigue and brittle fracture failures. Fatigue failure can occur in a relatively small number of load cycles, particularly in certain types of bridge connections and details. Brittle fracture failures have occurred in civilian structures, and are most likely to occur in frigid environments, especially in structures subjected to heavy impact loading.

Implementation

The recommended criteria are based on modifications of existing (AISC and AASHTO) allowable stresses for permanent structures and of the military design loads and design procedures given in TM 5-312. Hence, the recommended criteria are intended for use by bridge design offices rather than by combat engineers in the field. (See Chapter 4 for criteria for bridges designed by combat engineers.) Thus, the criteria "package" required by a bridge design office would consist of these criteria, which in turn require the use of modified TM 5-312 design loads and procedures and modified AISC or AASHTO allowable stresses.

Fabrication, Assembly, and Erection

Because the safety of a bridge can be reduced significantly by low-quality fabrication, assembly, and erection practices, the recommended criteria are based on the assumption that fabrication, assembly, and erection quality and tolerances are comparable to U.S. permanent construction. Fabrication, assembly, and erection shall be in accordance with TM 5-744. Weld quality shall be in accordance with the latest American Welding Society (AWS) requirements.[11] The recommended allowable stresses shall be reduced for lower quality fabrication, assembly, or erection. Field welding and field riveting of T/O structures are not recommended; bolting shall preferably be used. The necessary precautions during erection shall be taken, including provision of adequate support and bracing to protect against buckling and other erection problems.

Serviceability

The recommended criteria are based on limit state strength only. Serviceability requirements, including deflection and permanent set (yielding under service loads) are *not* covered by the criteria. (Deflection requirements are covered in Paragraph 6-4, TM 5-312.)

Loads, Moments, and Forces*

The procedures given in TM 5-312 for determining the design loads, moments, and forces used in the design of bridge members and fasteners shall be used, except as modified herein.

Two load cases shall be used in the design process. They are described in TM 5-312 (Table 4-3 "Types of Crossings," p 4-13) and consist of the *normal* vehicle crossing load case and the *caution* crossing load case. Normal crossings are convoy(s) consisting of vehicles not exceeding the posted bridge class. A caution crossing consists of a single line of vehicles crossing a one- or two-lane bridge on the bridge centerline. Each vehicle is spaced at 150 ft or more and shall not be more than 1.25 times the normal posted bridge class. Additional details are given in Section IV, Chapter 4, of TM 5-312 and Paragraph 2-54 of FM 5-36.[12]

Although the current procedure in TM 5-312 proportions all members, fasteners, and connections on

[11] *Structural Welding Code*, D1.1-75 (American Welding Society [AWS]).

*In certain cases, unrealistically low loads, moments, and forces can result if the current procedures in TM 5-312 are used.

[12] *Route Reconnaissance and Classification*, FM 5-36 (Department of the Army, May 1965).

the basis of the normal class loading, the procedure recommended here requires that they be proportioned on the basis of either the normal or caution load cases, as specified.

Flexural Members and Flexural Splices—Designing for Bending or Lateral Torsional Buckling in Stringers

The live load moments corresponding to normal class crossings found in Appendix D of TM 5-312 shall be used to proportion the flexural members or flexural splices for bending and lateral torsional buckling.

The current procedure for designing steel stringers given in Paragraph 6-5 of TM 5-312 shall be used, except that the recommended formulas in Tables 1 and 2 of this report shall be used to determine the effective number of stringers.

Shear in Stringers and Shear, Compression, or Tension in Stringer Connection Fasteners

The live load shear force in stringers and shear, compression, or tension in stringer connections shall be determined by using the larger force resulting from the normal or caution load cases. The shear for single-lane bridges is based on the caution load case, whereas for two-lane bridges it is based on either the normal load case or, if larger, the caution load case. The following revision to Paragraph 6-6, TM 5-312, shall be used to determine the shear force in stringers and the shear, compression, or tension in stringer connection fasteners:

6-6 Shear Check (Shear Design)

a) *Steel Stringer Bridges*

(2) *Live Load Shear.* The maximum shear loading for one stringer (v_{LL}) occurs when the vehicle is near the abutment or support. The value of the total live load shear in kips (V_{LL}) is obtained from the shear curves of Appendices D-4 and D-5 in TM 5-312. The value of v_{LL}, the total live load shear per stringer, including a 15 percent increase for impact, is:

$$v_{LL} = 1.15\, v'_{LL} \qquad \text{[Eq 1]}$$

where v_{LL} = total live load shear per stringer in kips, including impact

v'_{LL} = live load shear force per stringer in kips from Table 3, excluding impact.

Based on the above, the total design shear is:

$$v = \frac{V_{DL}}{N_s} + v_{LL} \qquad \text{[Eq 2]}$$

[Revised Eq 6-14b, TM 5-312]

where v = total design shear per stringer in kips

V_{DL} = total dead load shear in kips

N_s = total number of stringers in bridge.

Tension Members, Axially Loaded Compression Members, and Associated Fasteners and Connections

The forces in tension members, axially loaded compression members, and their associated fasteners and connections shall be the larger of the forces produced by the normal or caution crossing load cases.

Impact

The procedure to account for impact given in TM 5-312 shall be followed.

Allowable Stresses—Static Load Case

Members

Stringers—Bending and Lateral Torsional Buckling. The temporary* allowable bending stress F_{bt} for tension and compression on extreme fibers of members (compact) satisfying Section 1.5.1.4.1** of the 1969 AISC specification is:

$$F_{bt} = 0.83\, F_y \qquad \text{[Eq 3]}$$

where F_y = minimum specified yield stress.

The temporary allowable bending stress F_{bt} for tension and compression on extreme fibers for all other members (including flexural splices) is:

$$F_{bt} = 1.20\, F_b \qquad \text{[Eq 4]}$$

*"Temporary" refers to a 2- to 5-year design life.

**The moment redistribution provision in AISC 1.5.1.4.1 shall not be permitted.

Table 1

Recommended and Current TM 5-312 Equations for Determining the Effective Number of Stringers

Current TM 5-312 Equations		Recommended Equations
Single-lane: $N_1 = \frac{5}{S_s} + 1$	(Eq 6-7a)	Single-lane: $N_1 = c[\frac{5}{S_s} + 1]$
Two-lane: $N_2 = \frac{3}{8} N_s$ or $N_2 = N_1$	(Eq 6-7b)	Two-lane: $N_2 = c[\frac{3}{8} N_s]$ or $N_2 = N_1$
whichever is smaller.		whichever is smaller.

where N_1 = effective number of stringers for single-lane bridges
S_s = center-to-center stringer spacing in feet
N_s = number of stringers
N_2 = effective number of stringers for two-lane bridges
c = reduction factor given in Table 2.

Table 2

Values of Reduction Factor c* Used in Recommended Formulas for Determining the Effective Number of Stringers (Table 1)

	Ratio of Bridge Floor Width (out-to-out) to Bridge Span Length (W/L)	
Bridge Deck Type	**W/L ≤ 1.0**	**W/L > 1.0**
Glued-laminated timber or concrete	1.0	0.75
Nailed-laminated timber, plank, or multiple-layered	0.90	0.70

*The factor c accounts for the reduction in lateral load distribution when using nailed-laminated timber, plank, or multiple-layered decks and/or bridges which are very wide compared to their span length.

where F_b = the allowable bending stress as specified in Table 1.7.1 of the 1973 AASHTO specifications.*

For members not satisfying Section 1.5.1.4.1 of AISC, the requirements which cover compressive bending stress and bracing length requirements given in Table 1.7.1 of AASHTO are recommended to replace the corresponding requirements given in Paragraph 6-9 of TM 5-312.

Shear in Stringers. The allowable temporary shear stress F_{vt} on the gross** section of stringers is:

$$F_{vt} = 0.36\ F_y \qquad \text{[Eq 5]}$$

Axially Loaded Compression Members. The allowable temporary compressive stress F_{at} shall be determined by modifying the allowable stress F_a of the 1973 AASHTO specification as follows:

$$F_{at} = c_1 F_a \qquad \text{[Eq 6]}$$

where F_a = AASHTO allowable stress for compression in concentrically loaded columns (Article 1.7.1 AASHTO)

and c_1 is defined by the following:

For A 36 steel (with minimum yield stress of 36 ksi)

$$c_1 = 1.23 - [0.0025\ (KL'/r)] \qquad \text{[Eq 7]}$$

provided that

$$L'/r \leq 120$$

where L′ = length of member in inches (Article 1.7.1 AASHTO)
r = least radius of gyration of member in inches (Article 1.7.1 AASHTO)
K = effective length factor defined in Article 1.7.134(2) of AASHTO.

For other steels with a minimum yield stress F_y (in ksi)

$$c_1 = 1.23 - [0.000415\ (KL'/r)\sqrt{F_y}] \qquad \text{[Eq 8]}$$

provided $F_y \leq 50$ ksi and $L'/r \leq 120$.

*In determining the AASHTO allowable stress F_b, the 20 percent increase in allowable stress provided in Footnote (1) of Table 1.7.1 of AASHTO shall not be permitted.

**Gross section for shear of rolled shapes loaded in the plane of the web shall be taken as the product of the overall depth and the thickness of the web.

Table 3

Value of Live Load Shear Force per Stringer, v'_{LL}, Excluding Impact

	v'_{LL} for Single Lane* (kips)	v'_{LL} for Double Lane** (kips)
Wheeled vehicle	$1.25[(0.5+\frac{S_s}{32})V_A+(\frac{V_{LL_W}-V_A}{N_1})]$	$(\frac{S_s-2}{S_s})V_A+(\frac{V_{LL_W}-V_A}{N_2})$
Tracked vehicle	$1.25(\frac{V_{LL_T}}{2})$	$(\frac{S_s-2}{S_s})V_{LL_T}$

where V_{LL_W} = wheeled vehicle shear in kips, as given in Appendices D-4 and D-7† of TM 5-312

V_{LL_T} = tracked vehicle shear in kips, as given in Appendices D-5 and D-7† of TM 5-312

V_A = the heaviest axle load in kips, as given in column 3† of Appendix D-1 of TM 5-312

S_s = center-to-center stringer spacing in feet

N_1 = effective number of stringers for single-lane bridge defined in Table 1

N_2 = effective number of stringers for two-lane bridge defined in Table 1.

*The coefficient of 1.25 is used to adjust shear from the normal crossing case to the caution crossing case.

**For the double lane bridge case, v'_{LL} shall be computed for both single and double lanes and the larger value of v'_{LL} shall be used.

†The entries in Appendix D-7 and column 3 of Appendix D-1 are given in tons, which must be converted to kips.

Tension Members. The allowable temporary tensile stress on the net section of tension members is 0.73 F_y.

Bearing on Projected Area of Bolts and Rivets. The allowable temporary bearing stress on the projected area of bolts and rivets is 2.00 F_y, where F_y is the minimum yield stress of the connected material.

Fasteners and Connections

Table 4 gives the recommended temporary allowable stresses for bolts and shop rivets. Table 5 gives allowable shear stresses for shop and field welds (fillet welds), and Table 6 provides allowable stresses for bolts and shop rivets under combined tension and shear.

Low-quality fabrication, assembly, and erection procedures, particularly as related to fasteners and connections, can significantly reduce overall bridge safety. Hence, bolting is preferred over welding; field welding is *not* recommended, but if it is used, adequate weld quality shall be maintained. Field riveting is not to be used.

Full-penetration butt welds or groove welds, whether subjected to tension or compression, shall be proportioned for the same allowable stresses as the base metals being joined. Partial-penetration groove welds subject to compression or tension parallel to the axis of the weld shall also be proportioned for the same allowable stresses as the base metals being joined. Partial-penetration groove welds subject to tension normal to the axis of the weld shall not be used. Weld quality shall be in accordance with AWS requirements.[13]

The use of high-strength bolts shall conform to the provisions of the specifications for structural joints using American Society for Testing and Materials (ASTM) A 325 or A 490 bolts as approved by the Research Council on Riveted and Bolted Structural Joints,[14] except as modified herein.

Installation of high-strength bolts shall be by the turn-of-the-nut method or by load-indicating washers. Installation of A 307 bolts shall be by the turn-of-the-nut method. The rotations for installation by the turn-of-the-nut method, after all bolts have been brought to a snug condition, shall be applied in accordance with Table 7.

[13] *Structural Welding Code*, D1.1-75 (AWS).

[14] *Specification for Structural Joints Using ASTM A 325 or A 490 Bolts*, Approved by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, February 4, 1976.

Table 4

Recommended Allowable Stresses* for Bolts and Shop Rivets

Fastener Type	Tension (ksi)	Shear Plate** $L^{\dagger} < 50$ in.†† (ksi)	Shear Truss+ (ksi)
A 307			
Shank++	23.7	17.6	16.0
Threads#	23.7	14.0	12.5
A 325			
Shank++	48.6	32.0	27.0
Threads#	48.6	24.7	21.0
A 490			
Shank++	59.4	43.2	36.8
Threads#	59.4	32.6	28.1
A 502			
Grade 1	33.0	23.0	19.6
Grade 2	41.8	28.0	26.0

*Based on nominal diameter.
**Flat-plate type connection.
†L = length between extreme fasteners measured parallel to line of tensile stress.
††For L > 50 in., reduce allowable shear stress by 20 percent.
+Double-plane truss type connection.
++Threads excluded from shear plane.
#Threads not excluded from shear plane.

Table 5

Recommended Allowable Shear Stresses for Shop and Field Welds (for AFCS Bridges)

Required Electrode	Shop Weld Allowable Stress (ksi)	Field Weld Allowable Stress* (ksi)
E60	20.7	12.6
E70	23.1	13.6
E80	25.2	15.6
E90	27.0	16.2

*The field engineer may deem it necessary to further reduce these values if unusually low-quality fabrication is expected.

To inspect for proper tightening of bolts installed by the turn-of-the-nut method, the inspecting wrench shall be applied to the tightened bolt and the torque necessary to turn the nut or head less than 5 degrees (approximately 1 in. at 12-in. radius) in the tightening direction shall be determined. This torque shall be greater than the bolt installation inspection torques in Table 8.

For all fastener and connection criteria not covered herein, the latest AASHTO criteria are recommended, provided the procedures in the

Table 6

Recommended Allowable Stresses* for Bolts and Shop Rivets Subjected to Combined Tension and Shear

Fastener Types	Recommended Allowable Stresses (ksi)
Bolts	
A 307	
Shank**	$F_t{}^{\dagger} = 42 - 1.9F_s{}^{\dagger\dagger} \leq 23.7; F_s{}^{\dagger\dagger} \leq 17.6$
Threads+	$F_t = 32 - 1.9F_s \leq 23.7; F_s \leq 14$
A 325	
Shank**	$F_t = 83 - 1.9F_s \leq 48.6; F_s \leq 32$
Threads+	$F_t = 65 - 1.9F_s \leq 48.6; F_s \leq 24.7$
A 490	
Shank**	$F_t = 112 - 1.9F_s \leq 59.4; F_s \leq 43.2$
Threads+	$F_t = 85 - 1.9F_s \leq 59.4; F_s \leq 32.6$
Shop Rivets	
A 502 Grade 1	$F_t = 44 - 1.6F_s \leq 33; F_s \leq 23$
A 502 Grade 2	$F_t = 56 - 1.6F_s \leq 41.8; F_s \leq 28$

*Based on nominal diameter.
**Threads excluded from shear plane.
†F_t = *allowable tensile stress on fastener in presence of shear stress*, F_s.
††F_s = shear stress on fastener.
+Threads not excluded from shear plane.

Loads, Moments, and Forces section of this chapter are used.

Design Criteria to Prevent Brittle Fracture

Brittle fracture is a type of catastrophic failure that usually occurs without prior plastic deformation and at extremely high speeds (as high as 5,000 ft/sec). The fracture is usually characterized by a flat fracture surface (cleavage) with little or no plastic deformation; it usually occurs at average tensile stress levels below those of general yielding. Although brittle fractures are not as common as fatigue, yielding, or buckling failures, when they do occur, they may be more costly in terms of human life and/or property damage; they can lead to catastrophic failures of structures with little or no prior warning and a consequent loss of load-carrying ability. In contrast, failures by general yielding in tension are preceded by considerable deformation, and the members can still carry loads. Thus, the consequences of failure by brittle fracture are much greater than those of general yielding; specific design precautions should be taken in T/O structures to avoid this type of structural failure.

Table 7

Nut Rotations for Installation of Bolts

Bolt Length	**Turn of Nut From Snug Condition***	
	Both Outer Faces of Bolted Parts Normal to Bolt Axis	**Both Outer Faces of Bolted Parts Sloped Not More Than 1:20 From Normal to Bolt Axis**
Up to and including 4 dia.	1/3 (120 degrees)	2/3 (240 degrees)
More than 4 but not exceeding 8 dia.	1/2 (180 degrees)	5/6 (300 degrees)
More than 8 but not exceeding 12 dia.	2/3 (240 degrees)	1 (360 degrees)

*"Snug" is the condition which insures that the parts of the joint are brought into good contact with each other.

Table 8

Suggested Torques for Bolt Installation Inspection*

Nominal Bolt Size (in.)	**Inspection Torque (ft-lb)**		
	A 307 Bolts	**A 325 Bolts**	**A 490 Bolts**
1/2	50	100	125
5/8	100	200	250
3/4	175	350	440
7/8	285	570	715
1	425	850	1,070
1 1/8	525	1,050	1,500

*These torques are average values based on the use of new bolts. Adjustments may be necessary if old or rusted bolts are used, or if bolts with a lubricant are supplied. In these instances the provisions of the specifications of the Research Council on Riveted and Bolted Structural Joints shall be followed to establish new inspection torque values.

While the number of brittle fractures in structures such as bridges or buildings is small, the overall safety and reliability of these structures can be improved significantly by rather small changes in material specifications, design considerations, and fabrication control.

As a general rule, the designer must properly proportion his structure to prevent failure by tensile overload (yielding or ductile fracture), compressive instability, and stable crack growth (for example, arising from fatigue) or unstable crack growth (brittle fracture). Design to prevent brittle fracture usually requires (1) using a relatively low allowable tensile stress; (2) the elimination (insofar as possible) of structural details that act as stress raisers and can be potential fracture initiation sites, e.g., weld flaws, mismatches, holes, intersecting plates, and arc strikes, etc.; and (3) the selection of suitable materials. Unfortunately, large, complex structures (welded or bolted) cannot be designed or fabricated without discontinuities, although good design and fabrication practices can minimize the original size and number of these discontinuities. It is realized that stress concentrations or discontinuities will be present, but the designer assumes that the structural materials will yield locally and redistribute the load in the vicinity of the stress concentrations or discontinuities. The selection of materials and allowable stress levels is based on the realization that cracklike discontinuities in large, complex structures may be present or may initiate under cyclic loading and that some level of notch toughness is desirable.

At low temperatures, the maximum stress levels for all primary tension load-carrying members and flexural members subjected to tensile stresses shall be the same as those previously developed in the **Allowable Stresses—Static Load Case** section, but reduced for low temperatures, in accordance with the factors given in Table 9. This reduced stress is referred to as the "service-temperature-adjusted" maximum allowable tensile stress. *The* reduction factors in Table 9 for a given service temperature range are based on the requirement that if repeated loadings (fatigue) cause a crack to propagate, the safety of the structure against brittle fracture will be approximately the same for all of the steels. However, if the stress range at the location in question is sufficiently low, a fatigue crack may not develop and the "service-temperature-adjusted" maximum allowable tensile design stress will provide a design less susceptible to brittle fracture. The three zone steel types shown in Table 9 conform to the S4 supplementary requirements of ASTM A 709,[15] which include three temperature zones for the AASHTO toughness requirements.[16]

Meeting these requirements will insure that the steels have a moderate level of notch toughness at the indicated temperatures and that if designed and fabricated properly, the structures should perform satisfactorily during their 5-year lifetime.

---

[15] *Standard Specification for Structural Steel for Bridges*, A 709-74 (American Society for Testing and Materials, 1974).

[16] *Standard Specifications for Highway Bridges* (American Association of State Highway and Transportation Officials, 1973).

Table 9

Temperature Reduction Factors for Maximum Allowable Static Tensile Design Stress at Low Temperatures

	Reduction Factors for Service Temperatures of			
Steel Type*	**−31°F to −60°F**	**−1°F to −30°F**	**+32°F to 0°F**	**above 32°F**
Zone III	1.0	1.0	1.0	1.0
Zone II	0.8	1.0	1.0	1.0
Zone I	0.6	0.8	1.0	1.0
General (no toughness control)	Not permitted	0.6	0.8	1.0

*The three zone steel types conform to the S4 supplementary requirements of ASTM A709-74.

Secondary structural members and primary compression load-carrying members need not meet the AASHTO notch toughness requirements, but shall meet all other ASTM[17] requirements for the steel.

Weld metals used to fabricate T/O structures shall meet the AASHTO toughness requirements of the steels joined. Weld-metal impact-test specimens shall be fabricated and tested in accordance with AWS testing procedures.[18] In addition, all applicable AWS requirements for the qualification of welding procedures shall be followed. Heat-affected zone notch-toughness specimens are not required for these steels.

Several general recommendations dealing primarily with the fabrication and use of T/O structures will improve the overall safety and reliability of such structures. These are:

1. Use bolted construction rather than welded construction wherever possible, especially in the field.

2. Avoid using sheared plates. Use proper procedures for flame cutting of plate edges, so that small edge irregularities are minimized.

3. Use multiple load-path structures such as stringer bridges with many individual beam members rather than large, deep box girders with fewer individual members. Failure of a single member of a multiple load path structure may not necessarily lead to failure of the structure.

4. For service temperatures less than 0°F minimize any impact effects such as dropping of loads or sudden stops of traveling vehicles. Because rate of structural loading affects the resistance to fracture of structural steels, lower loading rates are desirable.

5. Avoid using very thick plates. Thick plates increase the constraint ahead of any cracks and create a more *un*favorable stress condition.

6. Avoid using complex structural details that act as stress concentrations. Not only is the stress increased, but the probability of cracks growing from discontinuities is increased.

7. Use drilled holes for bolted construction, coupled with machined edge preparation to minimize the formation of small cracks that might result from sheared or flame-cut edges.

Design Criteria for Fatigue

In contrast to design for static loadings, which is in terms of loads and static load capacity, fatigue design requires consideration of (1) the details at which fatigue may control the design, (2) the stress ranges or loading frequency history to which these details will be subjected, (3) the number of cycles of loading to which the details will be subjected, and (4) the allowable fatigue stress range based on the preceding three factors.

In general, fatigue is taken into account by means of a fatigue check of the static design. If the fatigue requirements are not satisfied, the members must be modified or redesigned to satisfy the fatigue requirements as well as the static load requirements. The following section outlines the steps to be followed in a design check for fatigue.

---

[17] *Annual Book of Standards* (American Society for Testing and Materials. current edition).

[18] *Specification for Mild Steel Covered Arc-Welding Electrodes.* AWSA5.1(AWS).

Design Check Procedure for Fatigue

1. Fatigue-stress limitations

a. Identify the points of possible fatigue failure by the detail number shown in Figure 1 and listed in Table 10 (e.g., No. 5—end of cover plate, No. 7—point of stiffener attachment, etc.).

b. Define the load type to which the fatigue-sensitive detail will be subjected (Type I, II, III, or IV of Figure 2). If information is not available to determine the load type, use of load Type III is recommended.

c. Determine the number of cycles of loading expected during the life of the structure (< 50,000, < 100,000, < 500,000, or < 2,000,000) (Table 11).

d. Determine the maximum allowable fatigue design stress range on the basis of the values established in steps a, b, and c by

(1) Determining the load-type factor C_L from Table 12.

(2) Determining the maximum allowable fatigue design stress range S_r using Eq 9:

$$S_r = C_L S \qquad \text{[Eq 9]}$$

where S = base allowable stress range given in Table 13.

e. Determine the maximum allowable static tensile design stress for the details in question. In some instances, the type of steel available or to be used may necessitate reducing this stress for low service temperatures. See step f.

f. Select the appropriate temperature reduction factor from Table 9 for the steel type to be used, and apply it to the maximum allowable static tensile design stress (found in the **Allowable Stresses—Static Load Case** section) to obtain the "service-temperature-adjusted" maximum allowable static tensile design stress.

g. If appropriate, determine the maximum allowable static compressive design stress for the detail in question.

2. Fatigue-stress check

a. Determine the minimum stress, maximum stress, and range of stress to which the fatigue-sensitive detail in question will be subjected during the life of the structure. These member forces and stresses shall be determined in accordance with the requirements in the section on **Loads, Moments, and Forces.** (Minimum and maximum stresses are the algebraic values of stress.)

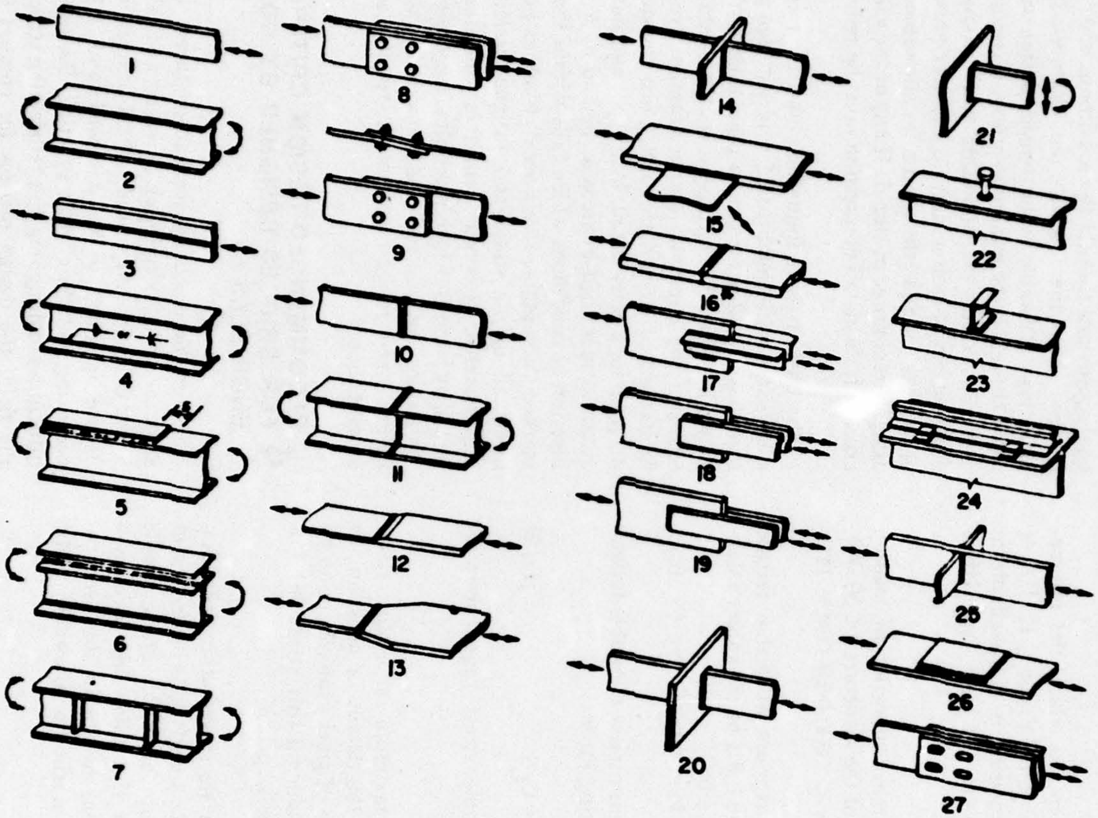
b. The fatigue resistance of the detail is adequate, provided that (1) the maximum tensile stress is less than the "service-temperature-adjusted" maximum allowable static tensile design stress of step f above, (2) the minimum stress, if compressive, does not exceed the allowable static compressive design stress of step g, and (3) the maximum range of stress is less than the maximum allowable fatigue design stress range of step d. If any of the values are exceeded, the detail in question must be redesigned.

The basic fatigue limitations and their relationships to the static and brittle fracture design requirements can be illustrated in a diagram[19] of the type shown in Figure 3, which presents the limitations in terms of the minimum and maximum stress. The limitations of steps d through g are shown in the diagram for a given detail, load type, and number of cycles as identified in steps a, b, and c. Any combination of conditions which falls within the region marked "acceptable stress area" (such as point A), meets all of the static and fatigue design stress requirements. However, conditions such as represented by point B, which meet all static design stress limitations but exceed the maximum allowable fatigue stress range requirement, require a design modification.

4 RECOMMENDED DESIGN CRITERIA FOR BRIDGES DESIGNED BY COMBAT ENGINEERS

Bridges designed by combat engineers (combat engineer bridges) are often designed and fabricated in the field. In comparison, AFCS bridges (Chapter 3) are intended to be designed and fabricated according to U.S. civilian practice. Because of the difficulties of designing and fabricating structures in the field, the design process for combat engineer bridges must be relatively simple, and the fabrication quality may be lower than that of AFCS bridges. It is recommended that special guidance be provided for the combat engineer in identifying and solving

[19] W. H. Munse, *Fatigue of Welded Steel Structures* (Welding Research Council, 1964).



*Detail No. 16 not recommended.

Figure 1. Structural details for fatigue design requirements. Details are described in Table 10. From *Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings* (AISC, 1969).

Table 10

Description of Structural Details for Fatigue Design

General Condition	Situation	Detail Example Nos. (Figure 1)*
Plain material	Base metal plate or rolled sections with rolled or cleaned surfaces.	1(1),** 1(2),** 2(1),† 2(2)†
Built-up members	Base metal and weld metal in members, without attachments, built up of plates or shapes connected by continuous fillet welds or full penetration groove welds parallel to the direction of applied stress.	3,4,6
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends.	5
	Calculated flexural stress at toe of welds on girder webs or flanges adjacent to welded transverse stiffeners.	7
Mechanically fastened connections	Base metal at net section of high-strength bolted connections, except bearing-type connections subject to stress reversal, and axially loaded joints which induce out-of-plane bending in connected material.	8
	Base metal at net section of other mechanically fastened joints.††	9(1)
	Shear on fasteners of mechanically fastened joints.††	9(2)
Groove welds	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2½, when reinforcement is not removed.	10,11,12,13
	Base metal or weld metal in or adjacent to full penetration groove welds in tee or cruciform joints.	14
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading.	15
	Weld metal of partial-penetration transverse groove welds, based on effective throat area of the weld or welds.	16 +
Fillet-welded connections	Base metal at ends of intermittent fillet welds.	24
	Base metal at junction of axially loaded members with fillet-welded end connections. Welds shall be disposed about the axis of the member so as to balance weld stresses.	17,18,19(1), 20(1)
	Continuous or intermittent longitudinal or transverse fillet welds (except transverse fillet welds in tee joints) and continuous fillet welds subject to shear parallel to the weld axis in combination with shear due to flexure.	19(2),21
	Transverse fillet welds in tee joints.	20(2)
Miscellaneous details	Base metal adjacent to short (2-in. maximum length in direction of stress) welded attachments.	22,23,24,25
	Base metal adjacent to longer fillet-welded attachments (8-in. maximum length in direction of stress).	26
	Base metal at plug or slot welds.	27(1)
	Shear on plug or slot welds.	27(2)

*These examples are provided as guidelines and are not intended to exclude other reasonably similar situations. Details Nos. 9(1), 19(1), 20(1), and 27(1) provide for stress in the base metal; 19(2), 20(2), and 27(2) provide for stress on the throat of the weld; and 9(2) provides for shear on the fasteners.

**For base metal (Detail No. 1, Figure 1), 1(1) and 1(2) refer to minimum yield strengths of 36 and 50 ksi, respectively.

†For rolled sections (Detail No. 2, Figure 1), 2(1) and 2(2) refer to minimum yield strengths of 36 and 50 ksi, respectively.

††Where stress reversal is involved, use of A 307 bolts is not recommended.

+This detail not recommended.

Load Type	Load Description	C_L
I	Primarily light weight vehicle crossings—50 percent of vehicles crossing weigh less than 0.3 of the maximum permitted vehicle weight.	1.90
II	Primarily medium weight vehicle crossings—50 percent of vehicles crossing weigh more than 0.5 of the maximum permitted vehicle weight.	1.35
III	Primarily heavy weight vehicle crossings—50 percent of vehicles crossing weigh more than 0.7 of the maximum permitted vehicle weight.	1.00
IV	All vehicle crossings are the maximum permitted vehicle weight.	0.75

*If sufficient information to determine the load type is not available, use Load Type III.

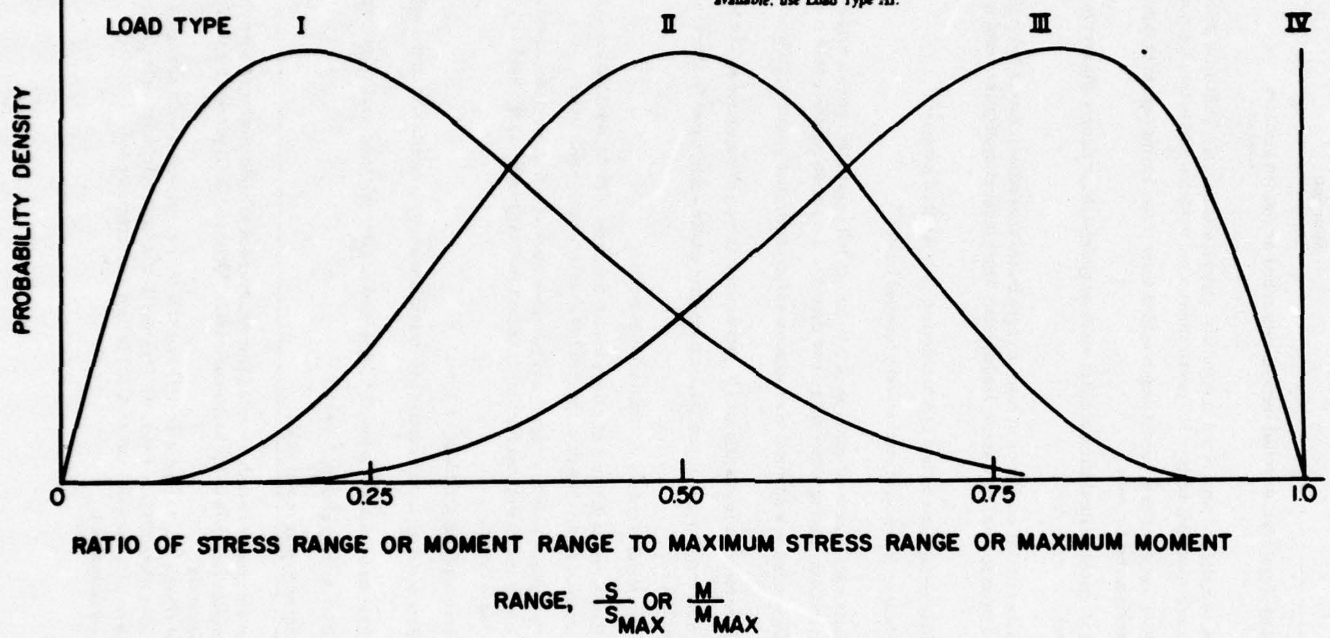


Figure 2. Loading frequency distributions used in fatigue design.

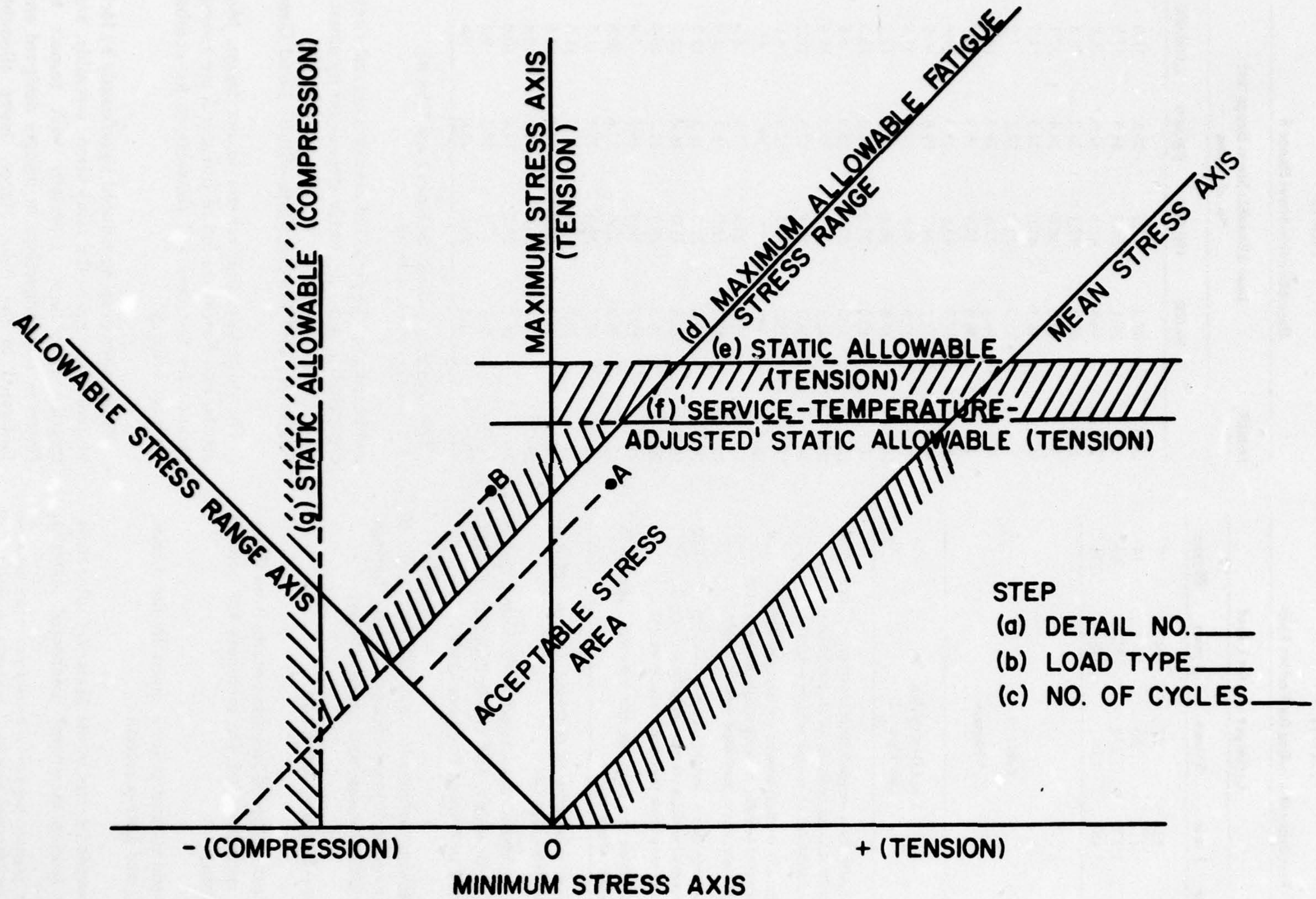


Figure 3. Modified Goodman fatigue diagram showing the basic fatigue design limitations.

Table 11

Frequency of Loading for Various Lives

	Cycles per Day for Life of			
No. of Cylces	**1 year**	**5 years**	**10 years**	**20 years**
50,000	137	27	14	7
100,000	274	55	27	14
500,000	1,370	274	137	69
2,000,000	5,480	1,096	548	274

Table 12

C_L* Factors

Load Type	Load Description (See Figure 2)	C_L
I	Primarily light weight vehicle crossings—50 percent of vehicles crossing weigh less than 0.3 of the maximum permitted vehicle weight.	1.90
II	Primarily medium weight vehicle crossings—50 percent of vehicles crossing weigh more than 0.5 of the maximum permitted vehicle weight.	1.35
III	Primarily heavy weight vehicle crossings—50 percent of vehicles crossing weigh more than 0.7 of the maximum permitted vehicle weight.	1.00
IV	All vehicle crossings are the maximum permitted vehicle weight.	0.75

*If sufficient information to determine the load type is not available, use Load Type III.

potential problems associated with lower quality fabrication, assembly, and erection practices which may be used in combat engineer bridges.

The following sections in Chapter 3 also apply to combat engineer bridges: **Scope**; **Safety, Savings, and Cost Effectiveness**; and **Serviceability.**

Members—Static Load Case

The following procedures and criteria from Chapter 3 are recommended for members for combat engineer bridges:

1. Procedures and criteria given in the **Loads, Moments, and Forces** section.

2. Allowable stresses in the *Members* subsection. Note that tension members* (connected material) and bearing on the projected area of bolts and rivets are included under allowable stresses for members.

*The field engineer may deem it necessary to reduce the allowable stress for connected material if unusually low quality field fabrication is expected, such as flame-cut holes, etc.

Table 13

Base Allowable Stress Range S

Detail*	Base Allowable Stress Range (ksi) No. of Cycles			
	50,000	**100,000**	**500,000**	**2,000,000**
1(1)	45.3	42.2	35.8	31.0
1(2)	59.1	54.3	44.5	37.5
2(1)	46.8	42.1	32.8	26.5
2(2)	52.8	47.1	36.2	28.8
3	43.0	38.0	28.4	22.1
4	56.1	43.6	24.3	14.7
5	25.3	20.3	12.2	7.9
6	56.1	43.6	24.3	14.7
7	36.4	29.9	18.9	12.7
8	47.7	43.6	35.3	29.4
9(1)	27.1	24.7	19.8	16.5
9(2)	37.5	34.1	27.5	22.8
10	37.8	30.8	19.2	12.7
11	40.6	33.9	22.3	15.5
12	35.2	27.7	15.9	9.8
13	42.1	36.1	25.3	18.6
14	34.2	27.9	17.5	11.7
15	25.0	20.5	12.9	8.6
16	**	**	**	**
17	25.2	20.6	12.9	8.6
18	17.0	12.9	6.7	3.9
19(1)	23.9	21.3	16.3	12.9
19(2)	23.5	21.0	16.0	12.7
20(1)	36.5	29.1	17.2	10.9
20(2)	16.7	14.3	10.1	7.4
21	36.2	32.6	25.6	20.8
22	44.7	34.6	19.1	11.5
23	35.9	29.0	17.7	11.5
24	35.9	29.0	17.7	11.5
25	40.6	30.8	16.3	9.4
26	26.7	22.2	14.4	10.0
27(1)	20.1	17.4	12.3	9.1
27(2)	21.8	18.7	13.0	9.6

*For description of details see Figure 1 and Table 10.
**This detail not recommended.

Fasteners and Connections—Static Load Case

The procedures and criteria in the **Loads, Moments, and Forces** section in Chapter 3 are recommended for fasteners and connections for combat engineer bridges.

Bridge safety can be reduced significantly by lack of quality control in the fabrication, assembly, and erection procedures, especially with respect to fasteners and connections in bridges designed and fabricated in the field. Hence, lower allowable stresses are recommended, except for field welds, for

combat engineer bridges as compared to AFCS bridges (Chapter 3).

For static design, the most recent AASHTO criteria for all fasteners, connections, and details, except shop and field welds, are recommended. Table 14 gives the allowable shear stresses for shop and field welds (fillet welds) for combat engineer bridges. For lower quality fabrication, assembly, and erection, the allowable stresses for fasteners and connections shall be reduced as judged necessary by the field engineer.

Table 14

Recommended Allowable Shear Stresses for Shop and Field Welds for Combat Engineer Bridges

Required Electrode	Shop Weld Allowable Stress (ksi)	Field Weld Allowable Stress* (ksi)
E60	18.0	12.6
E70	21.0	13.6
E80	24.0	15.6
E90	27.0	16.2

*The field engineer may deem it necessary to further reduce these values if unusually low-quality field fabrication is expected.

Bolting is preferred over welding. Field welding is *not* recommended, but if it is used, adequate weld quality shall be maintained. Field riveting is not to be used.

Full penetration butt welds or groove welds, whether subjected to tension or compression, shall be proportioned for the same allowable stresses as the base metals being joined. Partial-penetration groove welds subject to compression or tension parallel to the axis of the weld shall also be proportioned for the same allowable stresses as the base metals being joined. Partial-penetration groove welds subject to tension normal to the axis of the weld shall not be used. Weld quality shall be in accordance with AWS requirements.[20]

The use of high-strength bolts shall conform to the provisions of the specifications for structural joints using ASTM A 325 or A 490 bolts as approved by the Research Council on Riveted and Bolted Structural Joints,[21] except as modified herein.

The installation and inspection of bolts shall be as specified in Chapter 3.

[20] *Structural Welding Code*, D1.1-75 (AWS).

[21] *Specification for Structural Joints Using ASTM A 325 or A 490 Bolts*, approved by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, February 4, 1976.

Fatigue and Brittle Fracture Design Criteria

In some instances, particularly when a design must be done under field conditions, it may not be possible or desirable to include a direct check for fatigue and brittle fracture in the design. In those instances, the following procedure, which includes reduction factors which provide for brittle fracture and an approximate adjustment for the effects of fatigue, shall be followed.

1. Determine the minimum service temperature to which the structure will be subjected.

2. The maximum allowable static tensile* design stress, as determined for members in combat engineer bridges under static load, shall be reduced in accordance with the factors given in Table 15.

3. Whenever possible, fillet-welded connections should be avoided when severe fatigue loadings (significant reversal of stress) are expected.

Table 15

Reduction Factors for Maximum Allowable Static Tensile Design Stress at Low Temperatures (Field Design)

Yield of Steel (ksi)	Reduction Factors for Minimum Service Temperatures‡ of		
	−1°F to −30°F	+32°F to 0°F	Above 32°F
36	0.60	0.80	1.00
50 (Low-alloy)	0.45	0.60	0.75
100	This type of steel shall not be used.		

‡Service temperatures between −31°F and −60°F require the use of ASTM A 709-74 Zone III type steel and design in accordance with the fatigue provisions of Chapter 3.

Because of the importance in fatigue and brittle fracture of the geometry of the details in a structure and of the stress range and temperature to which a detail is subjected, rather severe but simple adjustments are provided in Table 15. As a result, some details under some loading conditions will have increased resistance against fatigue and fracture; however, even details having the lowest fatigue resistance can be expected to resist at least 10,000 cycles of very severe loading (represented by Type III in Figure 2). Other details having higher fatigue resistance can be expected to provide longer lives.

*Refers to primary tension load-carrying members and flexural members subjected to tensile stresses.

Implementation in Existing Technical and Field Manuals

The three primary manuals used by combat engineers for temporary bridge design are FM 5-34, FM 5-35, and TM 5-312. The recommended static, fatigue, and brittle fracture criteria for members in combat engineer bridges can be incorporated into Chapter 6 and Appendix E-1 of TM 5-312.

Due to the emphasis on expedient bridge design in FM 5-34, it is suggested that the recommended criteria for members be incorporated into FM 5-34 as follows: the static criteria for bending and shear in stringers presented in Chapter 3 can be used to construct a table for stringer selection similar to Table 7-3 of Chapter 7 of the current FM 5-34. It would include the moment and shear capacity and maximum bracing spacing requirements. If desired, the reduction factors in Table 15, which account for fatigue and brittle fracture, can be incorporated in the stringer selection table. The recommended procedures in Chapter 3 (**Loads, Moments, and Forces** section) for determining the effective number of stringers and the shear force can be incorporated into Section II, Paragraph 7-5 of FM 5-34. Design criteria for steel tension and axial compression members are not currently provided in FM 5-34; hence, implementation of criteria for these members into FM 5-34 is not treated herein.

If desired, the recommended criteria for members can be incorporated into Chapter 14 of FM 5-35.

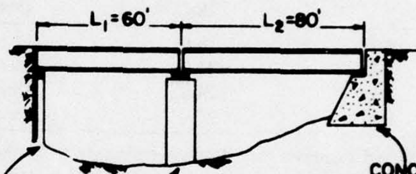
The recommended criteria for fasteners and connections can be incorporated into Appendix E-1 and Chapter 9 of TM 5-312 and Chapter 14 of FM 5-35. It should be noted that FM 5-34 does not currently provide design criteria for fasteners and connections for steel stringers.

5 DESIGN EXAMPLES

Static Load Case

The following is a revision of the example in Paragraph 6-11 of TM 5-312. It reflects use of the criteria in Chapters 3 and 4 and applies to both AFCS and combat engineer bridges.

Design of Steel Stringer Bridge Superstructure



Conditions Specified:

Minimum service temperature: 0° to 32°F

Steel type: $F_y = 36$ ksi

No ASTM toughness controls—see Tables 9 and 15

Figure 6-24 (TM 5-312): Design class 70, two-way bridge, A 36 steel stringers with timber nailed-laminated deck.

Remarks (References refer to TM 5-312 unless otherwise noted)

1. *Design Loads*

a. *Dead Load*

$$W'_{DL} = X + L\,(Y + ZC)$$ Equation 6-1

$$W'_{DL} = 800 + 80\,[3 + 0.2(70)]$$

$$W'_{DL} = 2160 \text{ lb/ft}$$

$$M_{DL} = \frac{W'_{DL}L^2}{8} = \frac{2.16(80)^2}{8}$$ Equation 6-2

$$= 1730 \text{ kip-ft}$$

$V_{DL} = \frac{W'_{DL}L}{2} = \frac{2.16(80)}{2}$	Equation 6-3
$= 86.4$ kips	

b. *Live Load*

$M_{LL_W} = 2450$ kip-ft	Appendix D-2
$M_{LL_T} = 2550$ kip-ft	Appendix D-3
$V_{LL_W} = 128$ kips	Appendix D-4
$V_{LL_T} = 127$ kips	Appendix D-5

Tracked vehicle controls for live load moment.

2. *Minimum Depth of Stringer*

$d = \frac{Lf_b}{69.0}$	Equation 6-4c
f_b = (temperature reduction factor) $(0.83\ F_y)$	Temperature reduction factor of 0.8 from Tables 9 or 15 of this report to account for the minimum service temperature range of 0° to 32°F.
$f_b = (0.80)(0.83)(36\text{ ksi})$ $= 23.90$ ksi	Base flexural stress for fully braced beam is 0.83 F_y (Chapter 3, Compact section, in accordance with Section 1.5.1.4.1 of AISC)
$d = \frac{(80)(23.9)}{69} = 27.7$ in.	All sections less than 27.7 in. in depth are eliminated.

3. *Stringer Selection*

a. *Minimum Number of Stringers*	Initially design for minimum number of stringers.
For Class 70, $W_R = 27$ ft 0 in. minimum	Figure 6-2 and Table 6-2

$$N_S = 6$$

b. *Effective Number of Stringers*

$S_s = \frac{W_R}{N_s - 1} = \frac{27.0}{6-1} = 5.4$ ft	Equation 6-5b
$N_2 = c[\frac{3}{8} N_s]$ or $N_2 = N_1$	Table 1 of this report.

whichever is smaller

$N_1 = c[\frac{S}{S_s} + 1]$ — Table 1 of this report.

$c = 0.90$ — This factor from Table 2 of this report corresponds to W/L < 1.0 and nailed-laminated timber deck.

$N_1 = 0.90[\frac{5}{5.4} + 1] = 1.73$

$N_2 = 0.90[\frac{3}{8}(6)] = 2.03$ or

$N_2 = N_1 = 1.73$

Use smaller of N_1 or N_2

Therefore: $N_2 = 1.73$

c. *Design Moment,* m

$m_{DL} = \frac{M_{DL}}{N_s} = \frac{1730}{6} = 288$ kip-ft — Equation 6-6

$m_{LL} = \frac{1.15 M_{LL}}{N_{1,2}}$ — Equation 6-8b

$m_{LL} = \frac{1.15(2550)}{1.73} = 1692$ kip-ft

$m = m_{DL} + m_{LL}$ — Equation 6-9a

$m = 288 + 1692 = 1980$ kip-ft

d. *Required Section Modulus*

$S_r = \frac{m(12)}{f_b}$ — Equation 6-10

$S_r = \frac{(1980)(12)}{23.9} = 994\ \text{in.}^3$

e. *Select Stringer (without cover plates)*

$S \geqslant S_r$ Use W 36 × 280 — AISC (p 2-7); section is compact with compression flange laterally supported in accordance with Section 1.5.1.4.1 of AISC.

$S = 1030\ \text{in.}^3$

4. *Shear Check*

a. *Live Load Shear* — Shear design procedure, Chapter 3, herein.

$v_{LL} = 1.15\, v'_{LL}$ — Eq 1 and Table 3 of this report.

Wheeled Vehicle Shear, Single Lane:

$$v'_{LL} = 1.25[(0.5 + \frac{S_s}{32})V_A + (\frac{V_{LL_W} - V_A}{N_1})]$$

Table 3 of this report.

$V_A = 21 \text{ tons} = 42 \text{ kips}$

Column 3, Appendix D-1

$V_{LL_W} = 128 \text{ kips}$

V_{LL_W} in Appendix D-4

$N_1 = 1.73$

From 3b. *Effective Number of Stringers* in this design example.

$$v'_{LL} = 1.25[(0.5 + \frac{5.4}{32})42 + \frac{128 - 42}{1.73}]$$

$$= 97.1 \text{ kips}$$

Wheeled Vehicle Shear, Double Lane:

$$v'_{LL} = (\frac{S_s - 2}{S_s})V_A + (\frac{V_{LL_W} - V_A}{N_2})$$

Table 3 of this report.

$$v'_{LL} = [(\frac{5.4 - 2}{5.4})42 + \frac{86}{1.73}] = 76.1 \text{ kips}$$

Tracked Vehicle Shear, Single Lane:

$V_{LL_T} = 127 \text{ kips}$

V_{LL_T} in Appendix D-5

$$v'_{LL} = 1.25(\frac{V_{LL_T}}{2}) = 1.25(\frac{127}{2})$$

Table 3 of this report.

$$= 79.4 \text{ kips}$$

Tracked Vehicle Shear, Double Lane:

$$v'_{LL} = (\frac{S_s - 2}{S_s})V_{LL_T} = (\frac{5.4 - 2}{5.4})(127)$$

Table 3 of this report.

$$= 80.0 \text{ kips}$$

Choose largest value of v'_{LL} for single- and double-lane and wheeled and tracked vehicles.

Therefore, $v'_{LL} = 97.1$ kips

The live load shear v_{LL} is:

$$v_{LL} = 1.15v'_{LL} = 1.15(97.1)$$

Eq 1 of this report.

$$= 111.7 \text{ kips}$$

b. ***Total Design Shear***

$$v = \frac{V_{DL}}{N_s} + v_{LL}$$

Eq 2 of this report.

$$v = \frac{86.4}{6} + 111.7 = 126.1 \text{ kips}$$

c. ***Shear Area Required***

$$A_{v_r} = \frac{v}{f_v} = \frac{126.1}{13} = 9.70 \text{ sq in.}$$

Equation 6-15
$f_v = F_{vt} = 0.36F_y = 0.36(36 \text{ ksi}) = 13$ ksi from Chapter 3 of this report.

Note that a W36 × 280 provides 32.3 sq in. of shear area.

Shear area based on gross section (product of overall depth [36.5 in.] and web thickness [0.885 in.]), AISC p 1-28.

32.3 sq in. >> 9.70 sq in.

Shear is not a problem.

5. ***Lateral Bracing***

a. ***Maximum Unbraced Length,*** L_c

For compact shapes:

L_c not to exceed $76b_f/\sqrt{F_y}$ nor

$20{,}000/[(d/A_f)F_y]$

AISC 1.5.1.4.1e

For W36 × 280, $b_f = 16.595$ in., $d/A_f = 1.40$

AISC pp 1-28, 1-29

$76b_f/\sqrt{F_y} = 210$ in. $= 17.5$ ft

$20{,}000/[(d/A_f)F_y] = 397$ in. $= 33.1$ ft

Therefore, $L_c = 17.5$ ft

6. See TM 5-312 for remaining design, including *Deck Design* and the check of the actual dead load versus the assumed dead load; appropriate changes in section modulus and shear area, if necessary, must then be made.

7. ***Rolled Beam with Cover Plate for 80-ft Span (Alternate Design)****

*The cover plate example illustrates only the cover plate design. Other design requirements, including shear, deflection, lateral bracing, deck design, and a check of actual dead load versus assumed dead load, must also be considered.

a. *Cover Plate Area Required*

$$A_{PL} = \frac{S_r - S}{d}$$ Eq 6-11b

$S_r = 994$ in.^{3} — S_r from 3.d. of this design example.

b. *Choose Rolled Beam*

Try W36 × 194: $S = 665$ in.^{3},

$d = 36.48$ in.,

$I = 12{,}100$ in.^{4} — I = moment of inertia

$$A_{PL} = \frac{994 - 665}{36.48} = 9.02 \text{ sq in.}$$

Try 9 in. × 1 in. plate, $A_{PL} = 9.00$ sq in.

Check S of cover plated section:

$S_{cp} = I/c = [12{,}100 + 2(A_{PL})y^2]/c$

y = distance from plate centroid to neutral axis = (36.48/2) + 0.5 = 18.74 in.

c = distance from top fiber to neutral axis = 19.24 in.

S_{cp} = section modulus of cover plated section

$S_{cp} = 18{,}421/19.24 = 957$ in.$^3 < 994$ in.^{3}

Since $S_{cp} < S_r$, try 10 in. × 1 in. plate.

$S_{cp} = (12{,}100 + 2(10)(18.74)^2]/19.24$

$= 994$ in.^{3}

$S_{cp} = S_r$ OK, Use 10 in. × 1 in. cover plate top and bottom on W36 × 194

Length of cover plate: 0.8 × span length — p 6-6

$= 0.80 \times 80$ ft $= 64$ ft

Fatigue Design Example (Cover Plated Beam)

The following is an example of a fatigue design check of the steel stringer bridge superstructure used in the previous example. The steps in this example are identified in the same manner as the check procedure presented in **Design Criteria for Fatigue** in Chapter 3.

1. *Fatigue—stress limitations*

a. The possible points of fatigue failure are at the ends of the partial-length cover plates (Detail No. 5) and at midspan (Detail No. 4 or 6). Figure 1 and Table 10 identify these details.

b. The loading type is specified as Type I (Figure 2).

c. The expected cycles of loading are specified as 500,000.

d. Based on steps a, b, and c, the maximum allowable fatigue design stress ranges from Equation

9 and Tables 12 and 13 are

Detail No. 5 = 23.2 ksi (= $C_L S$ = (1.90)(12.2) = 23.2)

Detail No. 4 or 6 = 46.2 ksi (= (1.90) (24.3) = 46.2)

e. The maximum allowable static tensile design stress, f_b = 29.88 ksi, from Chapter 3.

f. The "service-temperature-adjusted" maximum allowable static tensile design stress = 23.9 ksi, from Chapter 3.

g. If the compression flange is laterally supported, the compressive stress does not need to be checked.

2. *Fatigue stress check*

a. Minimum and maximum stresses

W36 × 194 beam with two 10 in. × 1 in. × 64 ft 0 in. cover plates.

End of cover plate (Detail No. 5).

Maximum stress:* $\dfrac{712.8'^{K} \times 12}{665} = 12.86$ ksi

(tension)

Minimum stress:** $\dfrac{103.68'^{K} \times 12}{665} = 1.87$ ksi

(tension)

Stress range = 10.99 ksi (= 12.86-1.87)

Midspan (Detail No. 4 or 6)

Maximum stress: $\dfrac{1980'^{K} \times 12}{994} = 23.90$ ksi

Minimum stress: $\dfrac{288 \times 12}{994} = 3.48$ ksi

Stress range = 20.42 ksi

b. Fatigue check

(1) Both maximum stresses are equal to or less than 23.9 ksi.

(2) Since minimum stress is tensile, compressive stress need not be considered.

(3) The stress ranges are less than the allowable 23.2 ksi at the end of the cover plate and 46.2 ksi at midspan.

Therefore, the fatigue resistance is adequate. If the loading had been Type IV, the allowable fatigue stress range at the end of the cover plate would have been 9.15 ksi (0.75 × 12.2, Tables 12 and 13), and the fatigue resistance of the stringer would *not* have been adequate at the end of the cover plate. To correct this, the cover plate would have to be extended beyond the point where the stress range is 9.15 ksi. The fatigue resistance of the midspan detail (No. 4 or 6) is also not adequate for Type IV loading, since the stress range of 20.4 ksi exceeds the allowable fatigue stress range of 18.2 ksi (0.75 × 24.3, Tables 12 and 13); this would require redesigning of the basic section.

6 EVALUATION OF EXISTING TM 5-312 DESIGN CRITERIA FOR STEEL STRINGER BRIDGES

This chapter evaluates existing TM 5-312 criteria for the static load case. Because fatigue and brittle fracture criteria are not covered in TM 5-312, they cannot be evaluated.

Bending in Stringers

Fully Braced Beams

For fully braced beams the current TM 5-312 criteria covering moment determination and allowable bending stress (0.83 F_y based on 1.5 × 0.55 F_y) should be adequate provided the following three conditions are satisfied:

1. The section meets the requirements of Section 1.5.1.4.1* of AISC.

2. The deck is solid (glued-laminated timber panels or concrete).

3. The ratio of the bridge floor width (out-to-out) to span length (W/L) is less than or equal to one.

*The 712.8 kip-ft moment of the end of the cover plate is based on a uniform load of 2.475 kips/ft, which is equivalent to the maximum dead plus live load moment at midspan of 1980 kip-ft.

**The 103.7 kip-ft moment at the end of the cover plate is based on a maximum load of 0.360 kips/ft, which is equivalent to the dead load moment at midspan of 288 kip-ft.

*Excluding the moment redistribution provision.

If one or more of these three conditions are not met, the TM 5-312 criteria are judged unconservative (i.e., not safe).

Lateral Torsional Buckling

The lateral torsional buckling requirements for compressive bending stress and bracing requirements given in Paragraph 6-9 of TM 5-312 appear to be unconservative. The criteria become even more unconservative when:

1. Nailed-laminated timber, plank, or multiple-layered decks are used.

2. Heavy wide flange shapes for stringers of about 200 lb/ft* or more are used.

3. The W/L ratio is greater than one.

Shear in Stringers

The current criteria in TM 5-312 for shear design in stringers, including the procedure to determine shear force and the allowable shear stress used, appear to be unconservative. The current shear force procedure in TM 5-312 is based on the normal crossing load case. Hence, the criteria become even more unconservative for the caution crossing load case.

Compression and Tension Members

The current criteria in TM 5-312 for compression and tension member design, including the determination of the compressive and tensile forces and the allowable stress used, appear to be unconservative. The compressive or tensile force used in design in TM 5-312 is based on the normal crossing load case. Thus, the criteria become even more unconservative for caution crossings on single-lane bridges. The unconservatism of the TM 5-312 criteria also increases for compression members having intermediate and high slenderness values.

Fasteners and Connections

Evaluation of the current TM 5-312 criteria for fasteners and connections depends on the actual force acting on the fastener and/or connection and the allowable stress used. Based on the current TM 5-312 design procedure, the actual force acting on the fastener will vary depending on the crossing type (normal or caution crossings), the number of lanes, and the type of member (flexural, tension, or compression) the fastener is connecting. As a result, a detailed evaluation was not performed. In addition, criteria for A 325 and A 490 high-strength bolts, the primary fasteners recommended in this report for T/O bridges, are not given in TM 5-312.

*Based on stringer depths fo 33 to 36 in.

7 CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The following conclusions were drawn from this study:

1. The current TM 5-312 criteria and procedures used in the design of T/O steel stringer highway bridges can result in bridges which are potentially unsafe and/or perform poorly.

2. The static, fatigue, and brittle fracture design criteria, procedures, and material specifications developed using state-of-the-art reliability-based methodologies should result in adequate safety and performance for T/O steel highway bridges. (See Volume II for details of the development and justification of the criteria presented in this volume.)

3. Material weight savings for steel stringers (standard rolled shapes under static load) designed using the criteria recommended in this report for T/O bridges rather than the permanent (AASHTO) bridge allowable stress with the recommended design procedures in Chapter 3 typically range from 15 to 20 percent.

4. Stringer weights obtained using the recommended criteria compare to those obtained using existing TM 5-312 criteria as follows:

a. Similar weights are obtained when concrete or glued-laminated timber decks are used in environments which are not cold.

b. Slightly increased weights are obtained when using the recommended criteria, if nailed-laminated timber, plank, or multiple-layered decks are used in environments which are not cold.

c. Significantly increased weights are obtained when using the recommended criteria for bridges in cold environments.

5. The criteria, design procedures, and material specifications are given in Chapter 3 for AFCS bridges. In Chapter 3, modified design loads and design procedures given in TM 5-312 and modified AISC or AASHTO allowable stresses are recommended; thus, the recommendations in Chapter 3 for AFCS bridges are intended to be used by bridge design offices.

6. The criteria, design procedures, and material specifications are given in Chapter 4 for bridges designed in the field by combat engineers. Thus, the recommendations in Chapter 4 can be incorporated into the Army manuals, including TM 5-312 and FM 5-34, which are used by the combat engineer in the field to design T/O bridges.

Recommendations

The following recommendations are made:

1. The design criteria, procedures, and material specifications given in Chapter 3 should be considered for use in the design of AFCS T/O bridges.

2. The design criteria, procedures, and material specifications for bridges designed in the field by combat engineers given in Chapter 4 should be considered for incorporation in TM 5-312, FM 5-34, and FM 5-35.

3. Consideration should be given to the development of procedures, criteria, and material specifications for design of glued-laminated timber decks for T/O steel stringer bridges.

REFERENCES

Annual Book of Standards (American Society for Testing and Materials, current edition).

Army Facilities Components System (AFCS)—Designs, TM 5-302 (Department of the Army, September 1973).

Army Facilities Components System (AFCS)—Logistic Data and Bills of Materials, TM 5-303 (Department of the Army, September 1973).

Army Facilities Components System (AFCS)—Planning, TM 5-301 (Department of the Army, September 1973).

Engineer Field Data, FM 5-34 (Department of the Army, December 1969).

Engineers' Reference and Logistical Data, FM 5-35 (Department of the Army, April 1971).

Military Fixed Bridges, TM 5-312 (Department of the Army, December 1969), with changes 1 and 2.

Munse, W. H., *Fatigue of Welded Steel Structures* (Welding Research Council, 1964).

Route Reconnaissance and Classification, FM 5-36 (Department of the Army, May 1965).

Specification for Mild Steel Covered Arc-Welding Electrodes, AWSA5.1 (American Welding Society [AWS]).

Specification for Structural Joints Using ASTM A 325 or A 490 Bolts, Approved by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, February 4, 1976.

Specifications for the Design, Fabrication and Erection of Structural Steel for Buildings (American Institute of Steel Construction [AISC], 1969), with supplements 1 (1970), 2 (1971), and 3 (1974).

Standard Specification for Structural Steel for Bridges, A 709-74 (American Society for Testing and Materials, 1974).

Standard Specifications for Highway Bridges, 11th ed. (American Association of State Highway and Transportation Officials [AASHTO], 1973).

Structural Design—Emergency Construction, TM 5-882-2 (Department of the Army, August 1963).

Structural Steelwork, TM 5-744 (Department of the Army, October 1968).

Structural Welding Code, D1.1-75 (AWS).

SI CONVERSION FACTORS

1 ft	= 0.3048 m	1 kip-ft	= 1.3558 kNm
1 ft-lb	= 1.3558 Nm	1 ksi	= 0.69 kN/cm^2
1 in.	= 2.54 cm	1 sq in.	= 6.4516 cm^2
1 kip	= 4.448 kN	(°F − 32) 5/9 = °C	

MSC

CERL DISTRIBUTION

Picatinny Arsenal
ATTN: SMUPA-VP3

US Army, Europe
ATTN: AEAEN

Director of Facilities Engineering
APO New York, NY 09827
APO Seattle, WA 98749

DARCOM STIT-EUR
APO New York 09710

USA Liaison Detachment
ATTN: Library
New York, NY 10007

US Military Academy
ATTN: Dept of Mechanics
ATTN: Library

Chief of Engineers
ATTN: DAEN-ASI-L (2)
ATTN: DAEN-FEE-A
ATTN: DAEN-FEB
ATTN: DAEN-FESA
ATTN: DAEN-FEZ-A
ATTN: DAEN-MCZ-S (2)
ATTN: DAEN-RDL
ATTN: DAEN-ZCP
ATTN: DAEN-PMS (12)
for forwarding to
National Defense Headquarters
Director General of Construction
Ottawa, Ontario K1A0K2
Canada

Canadian Forces Liaison Officer (4)
U.S. Army Mobility Equipment
Research and Development Command
Ft Belvoir, VA 22060

Div of Bldg Research
National Research Council
Montreal Road
Ottawa, Ontario, K1A0R6

Airports and Const. Services Dir.
Technical Information Reference
Centre
KAOL, Transport Canada Building
Place de Ville, Ottawa, Ontario
Canada, K1A 0N8

British Liaison Officer (5)
U.S. Army Mobility Equipment
Research and Development Center
Ft Belvoir, VA 22060

Ft Belvoir, VA 22060
ATTN: ATSE-TD-TL (2)
ATTN: Learning Resources Center
ATTN: Kingman Bldg, Library

US Army Foreign Science &
Tech Center
ATTN: Charlottesville, VA 22901
ATTN: Far East Office

Ft Monroe, VA 23651
ATTN: ATEN
ATTN: ATEN-FE-BG (2)

Ft McPherson, GA 30330
ATTN: AFEN-FEB

Ft Lee, VA 23801
ATTN: DRXMC-D (2)

USA-CRREL

USA-WES
ATTN: Concrete Lab
ATTN: Soils & Pavements Lab
ATTN: Library

6th US Army
ATTN: AFKC-LG-E

I Corps (ROK/US) Group
ATTN: EACI-EN
APO San Francisco 96358

US Army Engineer District
New York
ATTN: Chief, Design Br
Buffalo
ATTN: Library
Saudi Arabia
ATTN: Library

US Army Engineer District
Pittsburgh
ATTN: Library
ATTN: ORPCD
ATTN: Chief, Engr Div
Philadelphia
ATTN: Library
ATTN: Chief, NAPEN-D
Baltimore
ATTN: Library
ATTN: Chief, Engr Div
Norfolk
ATTN: Library
ATTN: NAOEN-D
Huntington
ATTN: Library
ATTN: Chief, Engr Div
Wilmington
ATTN: Chief, SAWCO-C
Charleston
ATTN: Chief, Engr Div
Savannah
ATTN: Library
ATTN: Chief, SASAS-L
Jacksonville
ATTN: Library
ATTN: Const. Div
Mobile
ATTN: Library
ATTN: Chief, SAMEN-D
ATTN: Chief, SAMEN-F
Nashville
ATTN: Chief, ORNED-F
Memphis
ATTN: Chief, Const. Div
ATTN: Chief, LMMED-D
Vicksburg
ATTN: Chief, Engr Div
Louisville
ATTN: Chief, Engr Div
Detroit
ATTN: Library
ATTN: Chief, NCEED-T
St. Paul
ATTN: Chief, ED-D
ATTN: Chief, ED-F
Chicago
ATTN: Chief, NCCCO-C
ATTN: Chief, NCCED-F
Rock Island
ATTN: Library
ATTN: Chief, Engr Div
ATTN: Chief, NCRED-F
St. Louis
ATTN: Library
ATTN: Chief, ED-D
Kansas City
ATTN: Library (2)
ATTN: Chief, Engr Div
Omaha
ATTN: Chief, Engr Div
New Orleans
ATTN: Library (2)
ATTN: Chief, LMNED-DG
Little Rock
ATTN: Chief, Engr Div
Fort Worth
ATTN: Library
ATTN: SWFED-D
ATTN: SWFED-F
Galveston
ATTN: Chief, SWGAS-L
ATTN: Chief, SWGCO-C
ATTN: Chief, SWGED-DC
Albuquerque
ATTN: Library
ATTN: Chief, Engr Div
Los Angeles
ATTN: Library
ATTN: Chief, SPLED-F
San Francisco
ATTN: Chief, Engr Div
Sacramento
ATTN: Chief, SPKED-D
ATTN: Chief, SPKCO-C
Far East
ATTN: Chief, Engr Div
Japan
ATTN: Library
Portland
ATTN: Library
ATTN: Chief, DB-6
ATTN: Chief, FM-1
ATTN: Chief, FM-2
Seattle
ATTN: Chief, NPSCO
ATTN: Chief, NPSEN-FM
ATTN: Chief, EN-DB-ST

US Army Engineer District
Walla Walla
ATTN: Library
ATTN: Chief, Engr Div
Alaska
ATTN: Library
ATTN: NPADE-R

US Army Engineer Division
Europe
ATTN: Technical Library
New England
ATTN: Library
ATTN: Laboratory
ATTN: Chief, NEDCD
North Atlantic
ATTN: Library
ATTN: Chief, NADEN
South Atlantic
ATTN: Library
ATTN: Laboratory
ATTN: Chief, SADEN-TC
Huntsville
ATTN: Library (2)
ATTN: Chief, HNDED-CS
ATTN: Chief, HNDED-SR
Lower Mississippi
ATTN: Library
ATTN: Chief, LMVED-G
Ohio River
ATTN: Laboratory
ATTN: Chief, Engr Div
ATTN: Library
North Central
ATTN: Library
Missouri River
ATTN: Library (2)
ATTN: Chief, MRDED-G
ATTN: Laboratory
Southwestern
ATTN: Library
ATTN: Laboratory
ATTN: Chief, SWDED-TG
South Pacific
ATTN: Laboratory
Pacific Ocean
ATTN: Chief, Engr Div
ATTN: FM&S Branch
ATTN: Chief, PODED-D
North Pacific
ATTN: Laboratory
ATTN: Chief, Engr Div

Facilities Engineer
FORSCOM
Ft Devens, MA 01433
Ft McPherson, GA 30330
Ft Sam Houston, TX 78234
Ft Carson, CO 80913
Ft Campbell, KY 42223
Ft Hood, TX 76544
Ft Lewis, WA 98433
TRADOC
Ft Dix, NJ 08640
Ft Monroe, VA 23651
Ft Lee, VA 23801
Ft Gordon, GA 30905
Ft McClellan, AL 36201
Ft Knox, KY 40121
Ft Benjamin Harrison, IN 46216
Ft Leonard Wood, MO 65473
Ft Sill, OK 73503
Ft Bliss, TX 79916
HQ, 24th Inf, Ft Stewart, GA 31313
HQ, 1st Inf, Ft Riley, KS 66442
HQ, 5th Inf, Ft Polk, LA 71459
HQ, 7th Inf, Ft Ord, CA 93941
West Point, NY 10996
ATTN: MAEN-E
Ft Benning, GA 31905
ATTN: ATZB-FE-EP
ATTN: ATZB-FE-BG
CAC&FL
ATTN: DFAE (3)
Ft Leavenworth, KS 66027
AMC
Dugway, UT 84022
USACC
Ft Huachuca, AZ 85613

AF/PREEU
Bolling AFB, DC 20332

AF Civil Engr Center/XRL
Tyndall AFB, FL 32401

Little Rock AFB
ATTN: 314/DEEE/Mr. Gillham

MSC

Naval Facilities Engr Command
ATTN: Code 04
Alexandria, VA 22332

Port Hueneme, CA 93043
ATTN: Library (Code LO8A)
ATTN: Morrell Library

Defense Documentation Center (12)

Washington, DC
ATTN: Bldg Research Advisory Board
ATTN: Library of Congress (2)
ATTN: Federal Aviation Administration
ATTN: Dept of Transportation Library
ATTN: Transportation Research Board

Engineering Societies Library
New York, NY 10017

USAES
Dept of Military Eng
Fixed Bridge Section
ATTN: CPT Phillips
Ft Belvoir, VA 22060

NAVFAC
HQ Engineering and Design
Structural Branch
Alexandria, VA 22332

Dept of Civil Engineering
ATTN: C. A. Cornell
Massachusetts Institute of Technology
Cambridge, MA 02319

Henry M. Reitz
Consulting Engineers
111 South Meramec
St Louis, MO 63105

Gregory Griffith
815 Graham St
Lynchburg, VA 24504

Case Institute of Technology
School of Engineering
Department of Solid Mechanics and Structural and Mechanical Design
ATTN: Fred Moses
Cleveland, OH 44106